

**EVALUATION OF SHANSEP  
PARAMETERS FOR SOFT  
BONNEVILLE CLAYS**

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# UDOT RESEARCH & DEVELOPMENT REPORT ABSTRACT

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16. Abstract  <p>This report contains a study of the SHANSEP parameters associated with samples of Bonneville clay taken near 3600 South and I-15 during and after the I-15 reconstruction project. A number of undrained <math>\overline{CK_0U}</math> shear tests were performed on samples taken at depths throughout the soil profile. These tests provided data with which to estimate the SHANSEP parameters. These parameters are given in the body of this report, and will be valuable to reference in future designs where similar Bonneville soil deposits are present.</p>			
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## EXECUTIVE SUMMARY

The purpose of this project was to determine the Stress History and Normalized Soil Engineering Properties (SHANSEP) parameters to characterize the undrained shear strength of soft Bonneville clay. Soil samples for this work were obtained near the MSE retaining wall near 3600 South on I-15 in Salt Lake City. The soil samples were obtained from a very soft clay layer between 18 and 20 ft deep. A series of constant rate of strain (CRS) consolidation tests and  $K_0$  consolidated undrained triaxial shear tests ( $\overline{CK_0U}$ ) were performed to determine these SHANSEP parameters.

Undrained shear strength in clays is a function of the soil type and structure, water content, stress history (over-consolidation ratio (OCR) and consolidation condition), and stress path during undrained loading. Classical analyses do not account for the effects of stress history and stress path in characterizing soil strength and in predicting field behavior. Stress history and stress path have very large effects on undrained strength of clays, leading to large errors in classical undrained analyses.

One approach, which accounts for the effects of stress history and stress path is the SHANSEP approach. The general idea behind the SHANSEP method is to perform a series of laboratory tests, which carefully control the stress conditions during consolidation, and control the stress path during undrained shear. These tests are performed over a range of stress histories and stress paths. The in situ stress history of the soil is then evaluated, and the stress path to which the soil will be imposed is determined. Then, strengths from the laboratory tests, which most closely replicate the field conditions, are used to predict the field behavior.

In the SHANSEP approach the following equation is used to describe the undrained shear strength of a soil subjected to a particular stress path:

$$\frac{S_u}{\sigma'_{v0}} = S \times (\text{OCR})^m,$$

where:  $S_u$  is the undrained shear strength,

$\sigma'_{v0}$  is the in situ effective vertical stress,

$S$  is the normally consolidated ratio of  $\left(\frac{S_u}{\sigma'_{v0}}\right)_{nc}$ ,

OCR is over consolidation ratio, and

$m$  is an exponent that usually falls between 0.75 and 1.0.

From this work, the following equation was found to predict the undrained shear strength of Bonneville clay in triaxial compression:

$$\frac{S_u}{\sigma'_{v0}} = 0.32 \times (\text{OCR})^{0.82}.$$

These results are based upon  $\overline{\text{CK}_0\text{U}}$  triaxial compression tests performed at OCR's from 1 to 6. These values of SHANSEP parameters are consistent, and in the range of values reported by other investigators for similar soils. The undrained shear strength for triaxial compression provides a close, but slightly conservative, estimation of the undrained shear for soils in a plane-strain, active condition.

The results of these tests were very consistent, and it was observed that normalized parameters very accurately describe the undrained shear strength and deformation behavior of these Bonneville clays. This indicates that SHANSEP analyses will provide good predictions of undrained field behavior, and will provide improved predictions of undrained soil behavior over classical approaches.

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## EVALUATION OF SHANSEP PARAMETERS FOR SOFT BONNEVILLE CLAYS

### 1.1 Introduction

The data presented in this report are intended to characterize the subsurface for the 3600 South area along the I-15 reconstruction project. The characterization will include a subsurface profile representing present in situ pressures, consolidation indices, and effective preconsolidation pressures ( $\sigma'_{v0}$ ). With preconsolidation pressures established, the undrained shear strength and the relationship between overconsolidation ratio (OCR) and the increase in undrained shear strength will be established. The relationship between OCR, preconsolidation pressure, and shear strength will be fit to the Stress History and Normalized Soil Engineering Properties (SHANSEP) equation (Ladd and Foote, 1974) that was developed by Charles Ladd of the Massachusetts Institute of Technology (MIT).

#### 1.1.1 Previous SHANSEP Data

Part of the site characterization plan of this study is the development of SHANSEP for the Bonneville deposit soils underlying the reconstruction of the I-15 through Salt Lake City near 3600 South. This work will follow the procedure outlined in Ladd in his 1974 publication. Work by Ladd in the Bonneville deposit was done in 1989. A follow up study was presented in Nicky Si Yan Ng 1998 thesis. Testing was done in  $K_0$ -consolidated undrained  $\overline{CK_0U}$  shear tests, as well as direct simple shear (DSS) tests. This report will add the  $\overline{CK_0U}$  results from this study to those of previous studies. The

Ladd and Ng studies were conducted on samples from 600 S. and 2400 S., respectively. The data presented in this report with respect to SHANSEP will be that from tests run on samples from 3600 S. The results of the  $\overline{CK_0U}$  tests were used to calculate the values of  $m$  for the prediction of shear strength according to the equation:

$$\frac{S_u}{\sigma'_{v0}} = S \times (OCR)^m, \quad (\text{Equation 1.1})$$

where:  $S_u$  is the undrained shear strength,

$\sigma'_{v0}$  is the in situ effective vertical stress,

$S$  is the normally consolidated ratio of  $\left(\frac{S_u}{\sigma'_{v0}}\right)_{nc}$ ,

OCR is over consolidation ratio, and

$m$  is an exponent that usually falls between 0.75 and 1.0 and is established by curve fitting.

### 1.1.2 Site Characterization Program

The testing program is aimed at characterizing the subsurface at 3600 South, I-15 corridor. The characterization will include in situ pressures ( $\sigma'_{v0}$ ), preconsolidation pressure ( $\sigma'_p$ ), Atterberg limits, grain size analysis and water content determinations to select the critical layer for the application of the SHANSEP method. Since the testing program for SHANSEP parameters is dependent upon the accurate measurement of the preconsolidation pressure, one dimensional consolidation testing is necessarily a preliminary step in this testing program.

## 1.2 Stress History and Normalized Soil Engineering Parameters

The complexity of the undrained shear behavior of soft clay is the motivation for developing a new design procedure for the stability of soft clays. Current design practice with regard to stability of soft clay subgrades is still largely dominated by the  $\phi=0$  method presented by Skempton (1948). Generally, this procedure combined with local experience and conservative factors of safety has produced safe designs. However, more recent research using commercially available triaxial testing systems with automated data collection, has improved model accuracy of the undrained strength behavior of clays. This improved model benefits from a theoretical framework to relate preconsolidation stress ( $\sigma'_{vc}$ ), overconsolidation ratio (OCR), and undrained shear strength ( $s_u$ ). This new theoretical framework, Stress History and Normalized Soil Engineering Properties (SHANSEP), was developed by Ladd and Foote (1974). Work done from the 1960's to the present has shown that some clays display normalized behavior, that is undrained shear strength behavior consistent when normalized by the confining stress. Figure 1.1 shows how the concept works. In the top curve, axial strain is plotted on the x-axis, and the deviator stress ( $\sigma_1 - \sigma_3$ ) is plotted on the y-axis. The lower curve is the same plot, however this time each deviator stress is normalized by the confinement pressure  $\sigma'_c$ . The data presented in this curve is an idealized, undrained shear strength for normally consolidated soils tested in triaxial compression.

Testing of normally consolidated soils is the first step in the triaxial testing necessary to develop SHANSEP, and is used to ensure that the clays being examined demonstrate “normalized” behavior. During this phase of testing, samples are consolidated onto the virgin portion of the consolidation curve, held for a creep period,

and then sheared. If the soil being tested is a normalized soil the undrained shear strength data would plot as shown in Figure 1.2. The next step in the SHANSEP testing process is to test at various overconsolidation ratios, and then plot the data according to equation 1.1.

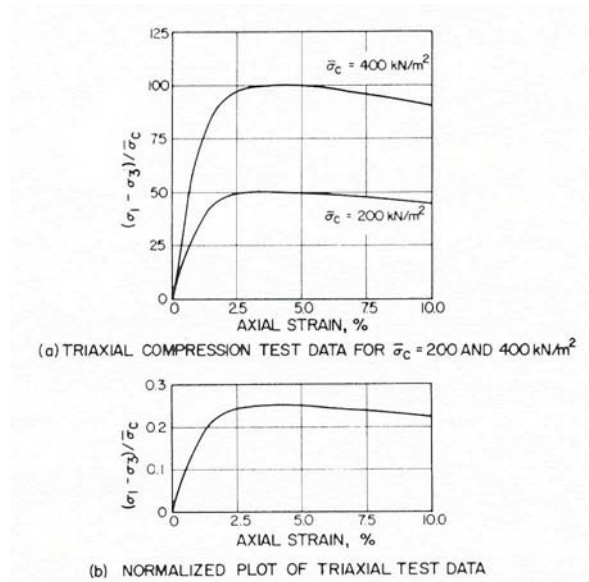


Figure 1.1. The Normalized Shear Strength Concept (after Ladd and Foote, 1974).

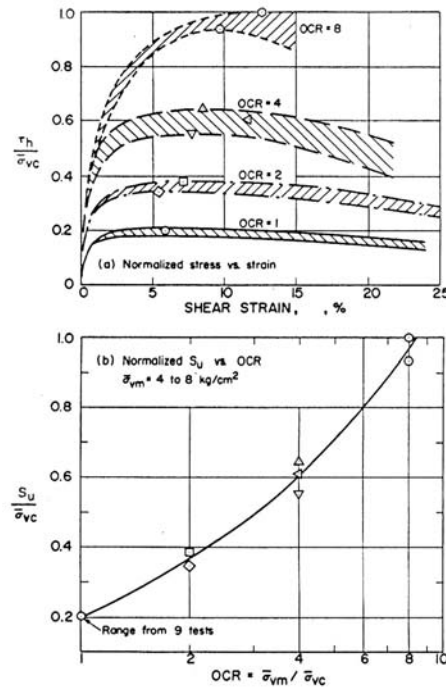


Figure 1.2. Normalized Undrained Shear Strength related to OCR. (After Ladd and Foote, 1974).

### 1.2.1 SHANSEP procedure

The procedure followed in this experiment follows that published by Ladd and Foote in 1974. A paraphrasing of the basic systematic procedure follows:

1. Select samples and, using one-dimensional consolidation testing, calculate the preconsolidation pressure ( $\sigma_{vo}$ ).
2. Using specimens from the same sample consolidate them to 1.5, 2.5 and 4.0 times the established  $\sigma_{vo}$ .
3. These tests should show a constant relationship between shear strength and consolidation pressure ( $s_u/\sigma_{vc}$ ), also seen as a  $c/p$  in soil mechanics literature. This should at least be true for the higher two pressures in the above step. If not, the SHANSEP procedure does not apply.
4. A pressure that shows a constant  $s_u/\sigma_{vc}$  relationship is selected as the laboratory consolidation pressure  $\sigma_{vm}$ .
5. The specimens are consolidated to this pressure and then allowed to swell to known overconsolidation ratios (OCR).

6. Shearing is initiated and the  $s_u/\sigma_{vc}$  ratio is plotted vs. OCR and this relationship is compared with existing data to ensure testing validity.

The advantages this special testing process provides are the ability to address questions regarding sample disturbance and stress path anisotropic behavior. Disturbance is minimized in this procedure by ensuring the specimens are loaded past the  $\sigma_{v0}$ , and onto the virgin portion of the consolidation curve. Stress induced anisotropy is addressed through the  $K_0$  consolidation portion of the triaxial  $\overline{CK_0U}$  compression testing. The at-rest principal stress relationship ( $\sigma_1/\sigma_2 = K$ ) is achieved by vertically consolidating the sample without allowing horizontal expansion. By controlling the flow into and out of the sample, along with continuously regulating the cell pressure, the triaxial testing apparatus maintains a constant cross section, thereby closely mimicking in situ consolidation conditions.

Strain rate effects have also been the topic of much research. Generally, it is accepted that different deposits respond differently to changes in strain rate. To address this concern a parametric study was undertaken to understand the scale of these effects on the Bonneville deposit, which covers much of the Salt Lake Valley. Strain rate effects influence both the preliminary one-dimensional consolidation testing, and the triaxial  $\overline{CK_0U}$  compression testing.

### 1.3 Testing Performed

A number of tests of various types have been performed in the course of this research. Many are addressed in the report concerning sample disturbance that has been

submitted (Bay et al., 2003). This report briefly describes the tests performed as they relate to this study.

### 1.3.1 Consolidation Testing

The results of the consolidation phase were vital to the completion of the triaxial ( $K_0$  consolidated) undrained shear test ( $\overline{CK_0U}$ ). The key piece of information to be gained from the CRS phase of testing is the preconsolidation pressure ( $\sigma'_p$ ). This preconsolidation pressure is the lowest pressure to which the samples must be loaded to ensure shear strength testing is done at a known overconsolidation ratio (OCR). OCR is defined by the equation:

$$OCR = \sigma'_{v \max} / \sigma'_{v \text{ test}}$$

where:

$\sigma'_{v \max}$  is the maximum vertical effective stress to which a specimen has been subjected, and

$\sigma'_{v \text{ test}}$  is the vertical effective stress during shear testing.

At pressures higher than the preconsolidation pressure, the compression of the specimen is on the “virgin” portion of the consolidation curve, and thus it is reasonable to assume that the specimen is under more pressure than it ever had been in situ. To test a specimen at a known OCR, it is placed under a stress that is known to be on the virgin portion of the consolidation curve. The pressure is then reduced and the sample is allowed to swell to a known OCR. The results of the triaxial  $\overline{CK_0U}$  tests are then used to

determine the  $m$  component of the SHANSEP equation, given in Equation 1.1. The testing for the new soil samples has two major components: one-dimensional consolidation testing and triaxial  $\overline{CK_0U}$  testing. The  $K_0$  portion of the  $\overline{CK_0U}$  test is a reference to the stress path along which the sample is consolidated. Loading along other stress paths is possible. For a detailed discussion of various stress path loadings and its implications refer to Holtz and Kovacs, (1981).

### 1.3.2 Index Properties

In both the 1999 exploration testing and the 2001 exploration testing, part of the testing program was to establish the Atterberg limits and the natural water contents of the samples at the various depths. The results for the samples used in the triaxial  $\overline{CK_0U}$

Table 1.1. Consolidation and soil properties for the SHANSEP specimens.

Boring	Depth (ft)	Sample Depth (ft)	$\sigma'_v$ (psi)	$\sigma'_p$ (Casagrande) (psi)	$\sigma'_p$ (Modulus) (psi)	$\sigma'_p$ error band (psi)	$w_n$ (%)
HS-1	17-19	18.2	11.8	23.0	24.0	22-24	61.0
HF-2	17-19	18.0	11.8	20.0	34.0	12-26	64.0
RS-3	17-19	17.9	11.8	22.0	21.0	17-24	67.0
RF-4	17-19	18	11.8	28.0	31.0	22-31	58.4

Table 1.1. Consolidation and soil properties for the SHANSEP specimens (continued).

Boring	PL (%)	LL (%)	$C_{CE}$	$\gamma_t$ (pcf)	$\gamma_d$ (pcf)	Grain Size %>75 $\mu$ m	Grain Size %2-75 $\mu$ m	Grain Size %<2 $\mu$ m
HS-1	26.0	36.0	0.259	101.6	62.7	3.3	61.7	35.0
HF-2	23.9	37.0	0.269	101.2	62.5	1.1	60.9	38.0
RS-3	25.5		0.419	101.1	61.2	5.2	60.8	34.0
RF-4	23.5	47.0	0.490	102.4	63.2	0.9	62.1	37.0

testing are shown in table 1.1. These are samples from the 17-19 ft. depth taken from the 2001 exploration.

Testing in triaxial  $\overline{CK_0U}$  was done on specimens from this level because of the high water contents, and the relatively high clay contents of this level in the profile. Results of the index testing are presented in Appendix A. The combination of conditions at this elevation makes this layer a lower limit to the shear strength to be found in this profile.

### 1.3.3 Triaxial Testing

Triaxial testing is used to develop the SHANSEP parameters. The in situ preconsolidation pressure is established from the CRS phase of the testing. Triaxial specimens are consolidated under  $K_0$  conditions to stresses higher than the in situ preconsolidation pressure to assure that the soil is in a normally consolidated state, then swelled to a known OCR, and finally sheared to failure. The resultant data is plotted and curve fitting is done to relate strength, OCR, and preconsolidation pressure according to equation 1.1.

The testing program for the development of SHANSEP parameters proceeds in the following basic steps:

1. Field Exploration
2. One dimensional Consolidation testing to establish preconsolidation pressures
3. Specimen selection
4. Triaxial shear testing on normally consolidated samples
5. Triaxial shear testing on overconsolidated samples.

The first three steps listed above are described in the sample disturbance report (UDOT Research Report UT-03.14) that has also been submitted. The triaxial testing on

normally consolidated samples (step 4 above) is done at multiples of the preconsolidation pressure as determined in the one dimensional consolidation step. Normally consolidated tests are run for samples consolidated to 1.5, 2.5, and 4.0 times the preconsolidation pressure, and the ratio of undrained shear strength ( $s_u$ ) to vertical consolidation pressure ( $\sigma'_{vc}$ ) is measured. Clay exhibiting normalized behavior will yield a constant value of the shear strength to vertical effective stress ( $s_u / \sigma'_{vc}$ ) at least for those consolidated to the higher vertical effective stresses. If ( $s_u / \sigma'_{vc}$ ) varies consistently with stress, the normalized soil parameters (NSP) concept does not apply to the clay (Ladd and Foote, 1974). Assuming the NSP concept does apply, testing proceeds with step five above to obtain ( $s_u / \sigma'_{vc}$ ) versus over consolidation ratio. In this step the minimum value of  $\sigma'_{vc}$  giving normalized behavior is used as the laboratory  $\sigma'_{vm}$ , and triaxial shear tests are performed at OCR values of  $2 \pm 0.5$ ,  $4 \pm 1$  and  $6 \pm 2$ . Results should then be checked against existing data to check reliability. The data points should form a smooth concave upward curve as shown in Figure 1.3.

The data in Figure 1.3 is collected from  $K_0$  consolidated direct simple shear tests. However, the curves from triaxial compression testing will have the same shape.

The data in this study is collected from specimens sampled at depths of 17-19 ft. during the 2001 site exploration. Specimens were selected from this depth because this layer had high water contents, and relatively low preconsolidation pressures, indicating that over consolidation due to desiccation was not a problem at this level. These factors, in addition to this layer having the highest clay percentages (see Table 1.1) in the profile, indicate that this level will provide a lower bound to the strength profile at this site.

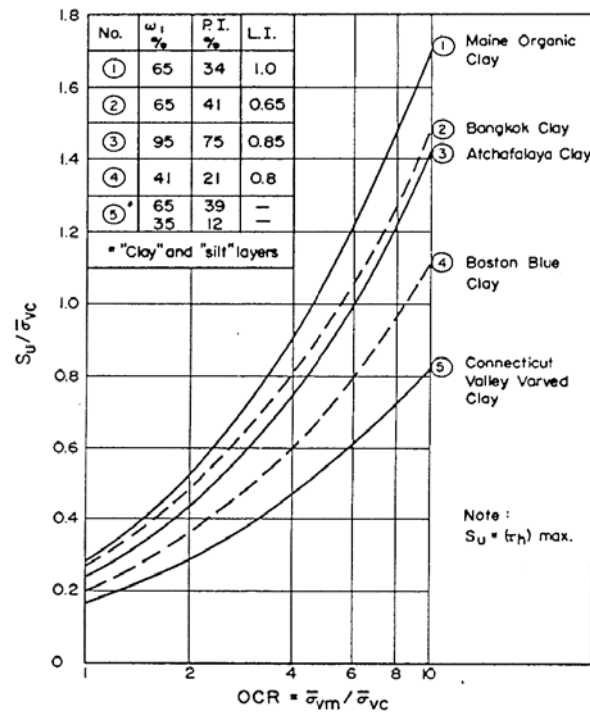


Figure 1.3. Variation of Normalized  $\overline{CK_0UDSS}$  DSS Strength Parameter with OCR for Five Clays (from Ladd and Foote, 1974).

#### 1.3.3.1 Sample Preparation

For triaxial testing, the sample preparation proceeds exactly as outlined in the sample disturbance report (UDOT Research Report UT-03.14) with variation in the length chosen and the way in which the sample was trimmed. A four inch section of tube is chosen from examining the radiographs, the soil sample is removed from the tube, and then the specimen is carefully trimmed with a wire saw as shown in Figure 1.4. Each sample is trimmed to a diameter of 1.4 inches (36 mm), and a height of 3 inches (76mm).

Special care is taken with each sample to ensure that the ends are square and parallel. This is important in order to avoid uneven loading and possible buckling in the triaxial apparatus.



Figure 1.4. Trimming the Triaxial Specimen.

The specimen is trimmed and weighed from which the total unit weight is calculated. After failure in triaxial compression, the specimen is dried and reweighed, and from this information natural water content and dry unit weight can be calculated. Placing a filter paper jacket around each triaxial specimen provides for increased radial drainage. An example of such a filter paper jacket is seen in Figure 1.5. Filter paper is also placed on each end between the filter stones and the specimen. The specimen is then placed on the triaxial base, and two latex membranes are installed using the brass tube and vacuum shown in Figure 1.6. The entire specimen assembly including the soil specimen, the filter papers, the porous stones, and the acrylic end caps are covered by the latex membranes, and sealed at the top and the bottom with two rubber o-rings. In this way the specimen and the pore water is isolated from the silicon oil that surrounds it and fills the cell.



Figure 1.5. Filter Paper Jacket to Assist Radial Drainage.



Figure 1.6. Installing a Latex Membrane on a Triaxial Specimen.

### 1.3.3.2 Triaxial Equipment Set-up

The equipment used in the triaxial portion of this testing program is the same as the CRS equipment with several variations. The additional pieces are the Trautwein Soil Testing flow pumps, as shown in Figure 1.7, and an internal load cell.

With the specimen installed on the triaxial base, the drainage lines are fitted into the specimen top cap. Unlike the single drained CRS test, the drainage lines utilized in the triaxial test allow for drainage at the top and bottom. The specimen in the triaxial compression test allow for drainage at the top and bottom. The specimen in the triaxial compression test is not contained in a rigid ring as is the CRS specimen. In the consolidation phase of the triaxial compression test, the specimen is kept from expanding

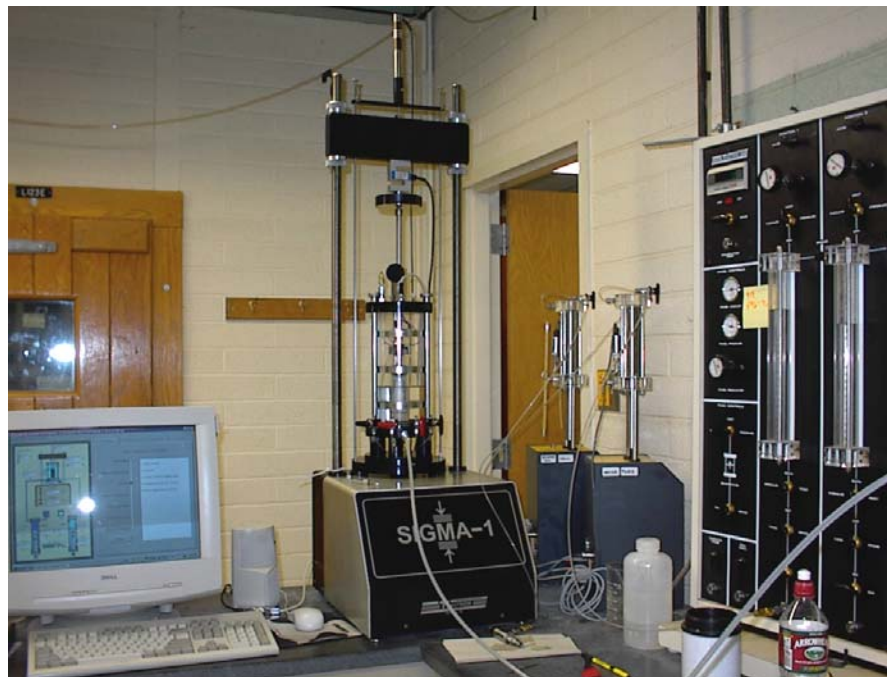


Figure 1.7. Sigma-1 Load Frame and Pore and Cell Pressure pumps just right of the Sigma-1. The Internal Load cell is also visible inside the triaxial cell.

by continuous pressure regulation by the pore and cell pumps. This pressure regulation simulates the rigid boundary used in the CRS test, thereby causing a one-dimensional consolidation, albeit with different drainage conditions than the CRS test. Positioning the acrylic cell wall, cell cap, and cell tensioning rods completes the cell assembly. The assembled cell is then placed in the Sigma-1 load frame, and the Trautwein Soil Testing True Path software is initialized from the PC start-up menu.

#### 1.3.3.3 Triaxial Operation

To begin operation of the triaxial apparatus, the software package is initialized from the start menu of the PC. The seating of the new specimen and assembly is the first action of each test. After the seating load has been placed on the sample, the triaxial cell is filled with oil. To fill the cell, a vent is placed in the top of the cell cap, the control panel is used to pressurize the vessel containing the oil, and the quick connect line from the oil tank to the triaxial cell is snapped into place. When the cell is completely full, the vent is removed, the pressure vented, and the quick connect line is removed.

Next, the cell is pressurized. The cell pump is connected to the cell, and the cell valve is opened. The pressure is selected somewhere between 5 and 10 psi. The pressure can be changed at any point, and several minutes should be allowed to let the cell pump close in on the exact position necessary to hold the selected pressure.

Pressurizing the cell necessarily pressurizes the specimen. With the specimen surrounded by a cell pressure in the range of 5 to 10 psi, the drain lines from the pore pump to the specimen can be flushed of air without additional disturbance

After the system is pressurized and saturated, it is necessary to allow time for the specimen to equilibrate to the new conditions. Allowing a certain length of time for this

to occur is vital to an efficient back-pressuring routine. All of the specimens in this study were given at least one hour to equilibrate during the maintain volume step. This amount of time provided good results in the backpressure routine.

#### 1.3.3.4 Backpressure

The software raises the pressure to the specimen according to measured pore stiffness. The volume is maintained by holding the difference between the cell and pore pressures constant at the same value as existed at the end of the seating routine. To ensure sample integrity, it is necessary to have an effective stress of at least 3.0 psi at the end of seating.

Evaluation of the effectiveness of the backpressure saturation is accomplished by observing the value of Skempton's B-coefficient. A B-coefficient of one indicates 100% saturation. In actual testing a B-coefficient of one is very unlikely to be reached. A coefficient of at least 0.98 achieved in two minutes or less was deemed an acceptable level of backpressure saturation, and when that was measured, the test was advanced to the consolidation phase.

Advancing from backpressure to consolidation is done manually. The maximum vertical stress and the loading rate are determined, and the consolidation phase is initiated. In the SHANSEP procedure (Ladd and Foote, 1974), specimens are consolidated along the  $K_0$  stress path to mimic in situ conditions, by replicating field anisotropic loading conditions. The  $K_0$  stress path is shown in Figure 1.8. In  $K_0$  consolidation, the sample begins to consolidate at point F on the figure. The software controls the pore pressure and the cell pressure to maintain a constant cross sectional area

in the specimen. In doing so, lateral expansion or contraction of the specimen is prohibited, and in this way stresses return to the at rest condition along the  $K_0$  line.

The description of the SHANSEP procedure indicates multiples of the preconsolidation stress should be 1.5, 2.5, and 4.0 times the vertical stresses applied to the normally consolidated samples. The normally consolidated specimens are loaded to the calculated multiples of the preconsolidation stress, allowed 24 hours to creep under constant vertical load, and then sheared. The procedure for shear testing at different OCR's is slightly different.

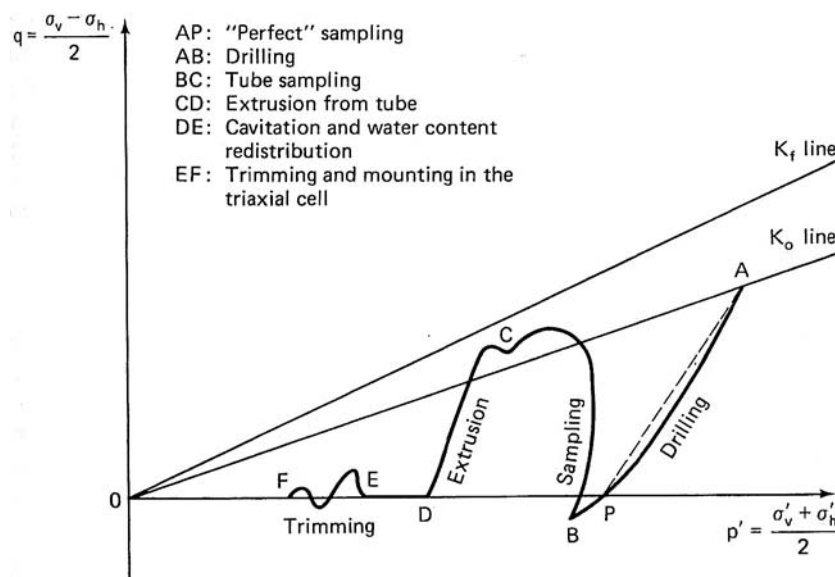


Figure 1.8. The  $K_0$  stress path with the stress path of a triaxial specimen superimposed (From Holtz and Kovacs, 1981)

To test specimens at OCR's of  $2.0 \pm 0.5$ ,  $4.0 \pm 1$ , and  $6.0 \pm 2$ , the lowest vertical stress from the normally consolidated samples displaying normalized behavior is taken as

the  $\sigma'_{vm}$ . Specimens are loaded to this stress and then allowed to swell under stresses reduced to the given OCR's. In this experiment, a  $\sigma'_{vm}$  value of 60 psi was selected. The specimens are allowed to creep for 12 hours, and then are sheared.

Shearing begins by ending the consolidation phase. The rate and limits of shearing with regard to limit strain, and limit vertical pressure are entered under the test data page. These parameters are set at the beginning of testing and can be changed at any time during testing. The specimens in this testing program were all limited to 15% strain, and 150 psi of vertical load.

The shearing of the specimens was generally the shortest phase of the testing. During shear, there are real time plots that can be monitored. These are:

1. Principal Stresses
2. Shear Stress
3. p-q
4. Stress Ratio
5. Pore Pressure.

From these real time plots the operator can get an immediate estimate of the shear strength peak and failure points. The shear strength peak occurs relatively quickly.

The triaxial  $\overline{CK_0U}$  test is complete at this point. During testing, the pore pressure, backpressure, cell pressure, vertical load, and pore volume change are continuously monitored. The final operator tasks are to breakdown the apparatus and take water content measurements for the failed specimen.

#### 1.4 Results

Data from the characterization testing are presented in tables in Appendix A for all of the specimens tested. Also presented in these tables are the preconsolidation

pressures as calculated by the Casagrande method, and the range of possible preconsolidation pressures as determined using the modified Casagrande method as presented by Holtz and Kovacs, (1981).

Results from the consolidation phase of the triaxial  $\overline{CK_0U}$  testing are presented in section 1.4.2 in the form of percentage axial strain vs. vertical effective stress, and from the shearing phase as shear stress vs. percentage axial strain. Data tables are provided to facilitate comparison of the results between the four boreholes, and further discussion is provided. Finally a plot of peak shear stress vs. overconsolidation ratio (OCR) is presented.

In section 1.4.4, the collected data are fit to a logarithmic equation describing the relationship between shear strength, consolidation pressure and OCR as presented by Ladd and Foote (1974), and the related plot is provided. A plot of the normalized shear strength vs. OCR is also provided.

#### 1.4.1 Characterization of the Subsurface Profile

The characterization of the subsurface profile for the site at 3585 South 500 West was based upon information collected from a sequence of tests to determine the values of distinguishing soil properties. This data is presented in tables in Appendix A.

The critical layer is that which is likely to have the lowest undrained shear strengths in the subsurface profile. The critical layer combines a high natural moisture content, low wet and dry densities, high percentages of fine particles, including clay particles, and a low preconsolidation pressure.

Referring to Appendix A, the 17 -19 ft. layer consistently has the highest moisture contents, lowest densities, and lowest preconsolidation pressures measured in each of the borings, while the fines content and clay content for this layer is similar to those of adjacent layers. The water contents measured at the 17 –19 ft. depth ranged from 58.7 to 67.0 percent. These natural water contents were 15 to 25 percent higher than those measured in other layers in the profile. This layer also had the lowest unit weights, ranging from 21 to 42 pcf less than other layers in the profile. The clay contents measured at the 17-19 ft. depth ranged from 34 to 38 percent and were generally consistent with clay contents measured throughout the profile. The combination of these characteristics distinguishes the 17-19 ft. depth as the layer likely to have the lowest shear strengths in the profile. Therefore, it was chosen as the critical layer.

Figure 1.9 shows how the in situ pressures ( $\sigma'_{vo}$ ) compare with the preconsolidation pressures ( $\sigma'_p$ ) as estimated from consolidation tests. In this figure, the 17-19 ft. level stands out as the least overconsolidated soil in the profile. It is interesting to note the difference in the values between  $\sigma'_{vo}$  and  $\sigma'_p$  at the 15 ft. depth. This difference likely indicates some desiccation has occurred to this depth. The  $\sigma'_p$  values were calculated by applying the Casagrande method (1948) to the consolidation curves.

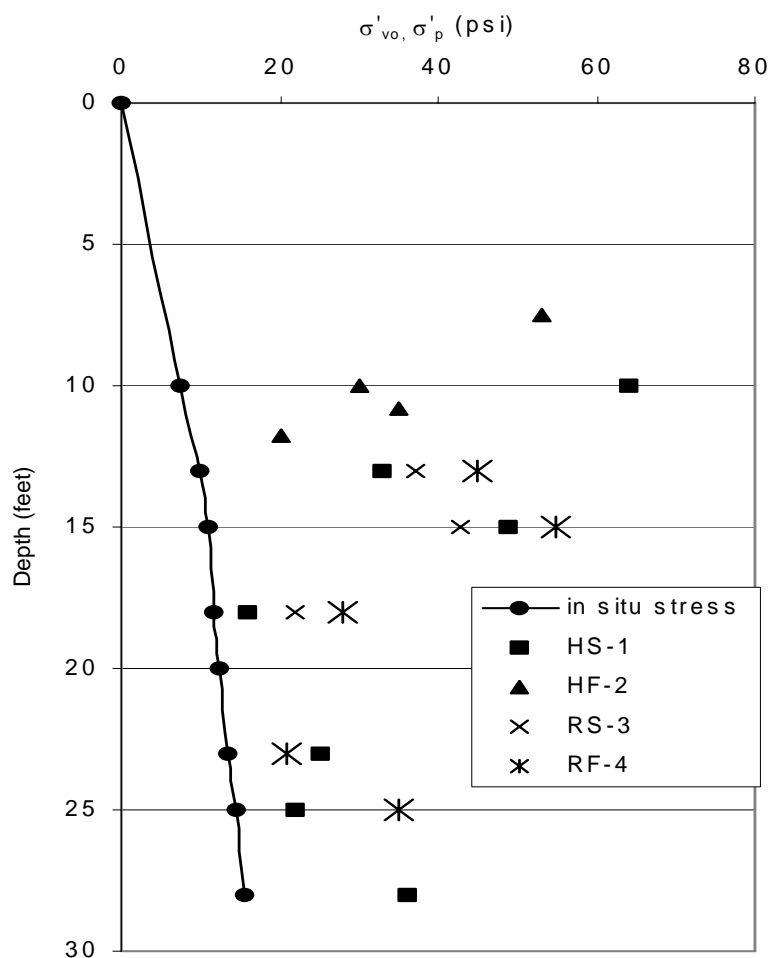


Figure 1.9. Subsurface pressures at 3585 South I-15, Salt Lake City.

Table 1.2 summarizes the same information as given in Appendix A for the samples tested in the 17 to 19 ft depth range. Listing the test results for the critical depth together allows a quick check on the consistency of the test results between the four different boreholes. An average sample for this layer has a liquid limit of 37, a plastic limit of 25, and a plasticity index of 12, which plots as ML or OL on the Unified Soil Classification Chart. Soil from the 17-19 ft. layer in this study is classified as OL based on color and odor observations. An average sample for this layer has a wet density ( $\gamma_t$ ) of

101.6 pounds per cubic foot, a dry density ( $\gamma_d$ ) of 61.4 pounds per cubic foot, a natural moisture content ( $w_n$ ) of 62.2%, and an in situ OCR of 2.

Figure 1.10 illustrates the similarity in compressibility of samples taken from the same depth in different boreholes.

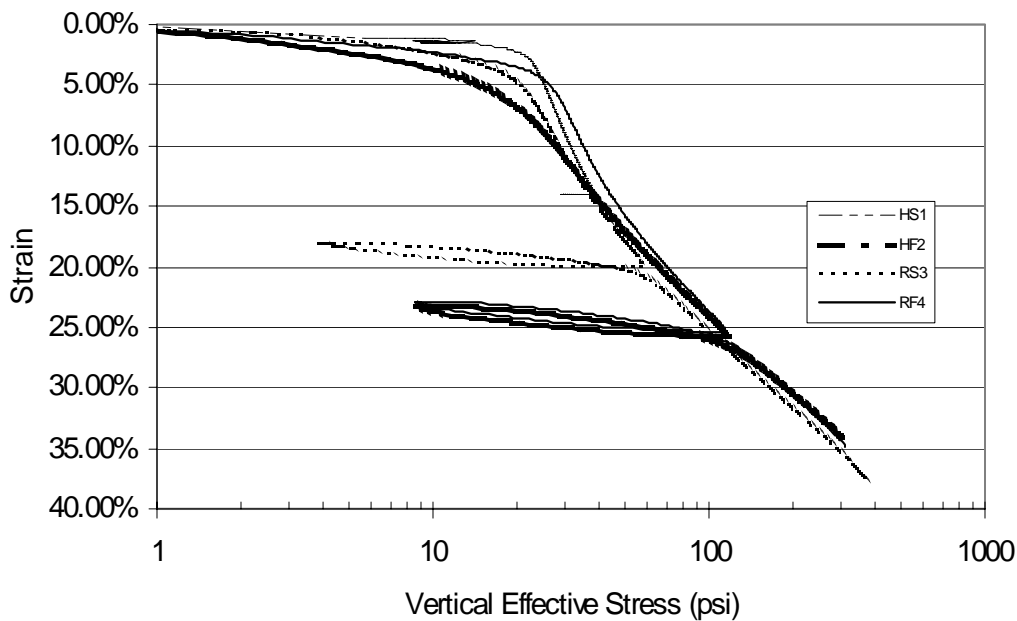


Figure 1.10. Superimposed Consolidation Curves from 2001 exploration at the 17-19 ft depth.

The close grouping of the curves gives visual indication of the narrow range of possible preconsolidation pressures. The consistency of the results in the CRS testing allowed some confidence in estimating the  $\sigma'_{v0}$  to be utilized in the  $\overline{CK_0U}$  portion of testing. The preconsolidation pressure was estimated to be 24 psi. This was on the upper end of the range of possible  $\sigma'_{v0}$  values for the Borings HS-1 and RS-3, and just slightly higher than the average of the Casagrande method values. This graphic also illustrates

the effect of the drilling and sampling methods on the estimation of the preconsolidation pressure, with the least disturbed samples demonstrating the sharpest bends and the highest possible preconsolidation pressures, as well as the most distinctive “S” shape to the consolidation curve.

#### 1.4.2 Consolidation Phase Results from Triaxial Tests

Figures 1.11 through 1.16 show the strain vs.  $\log p$  plots for the consolidation phase of each of the six  $\overline{CK_0U}$  tests performed on specimens from the 17-19 ft depths. The “tail” at the end of the consolidation curve represents the continuing straining occurring during the 24-hour creep period allowed at the end of the consolidation-loading phase. The creep period allows pore pressure dissipation. This feature is seen on each of the three tests conducted at an OCR of 1, Figures 1.11 through 1.13. In Figures 1.14 through 1.16, the portions of the curve shown at the end of the consolidation and creep periods represent the swelling period to allow undrained shear strength testing to be conducted at OCR’s of 2, 4 and 6. That means that the samples tested at the OCR’s have been consolidated to a vertical effective stress that is 2, 4 or 6 times as great as the vertical effective stress at which the shearing phase of the triaxial  $\overline{CK_0U}$  test begins. In this way the SHANSEP procedure addresses the question of strength gain characteristics for a given soil. This information would be important to a practitioner when planning staged embankment construction or a preloading scheme. Ultimately, the SHANSEP procedure provides an understanding of the relationship between strength gain and OCR

which will allow an accurate back calculation of the in situ shear strength based on in situ OCR's.

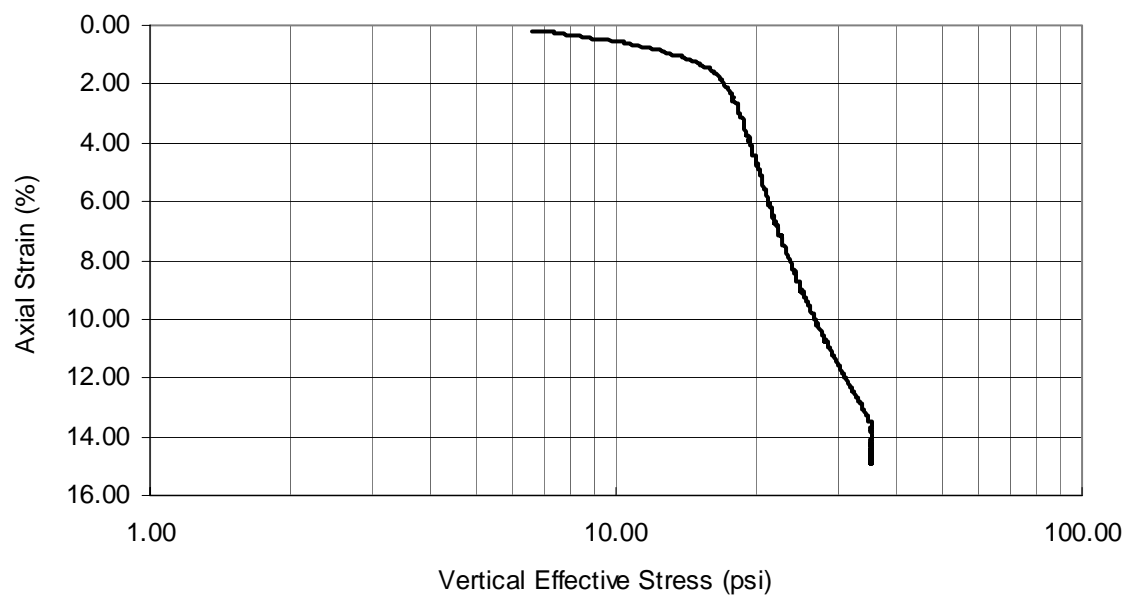


Figure 1.11. Consolidation curve for a sample from boring HS-1, depth of 18.1 ft.  $K_0$  consolidated to  $\sigma'_v$  equal to 1.5 times  $\sigma'_{v0}$ , which is 36 psi. OCR=1.

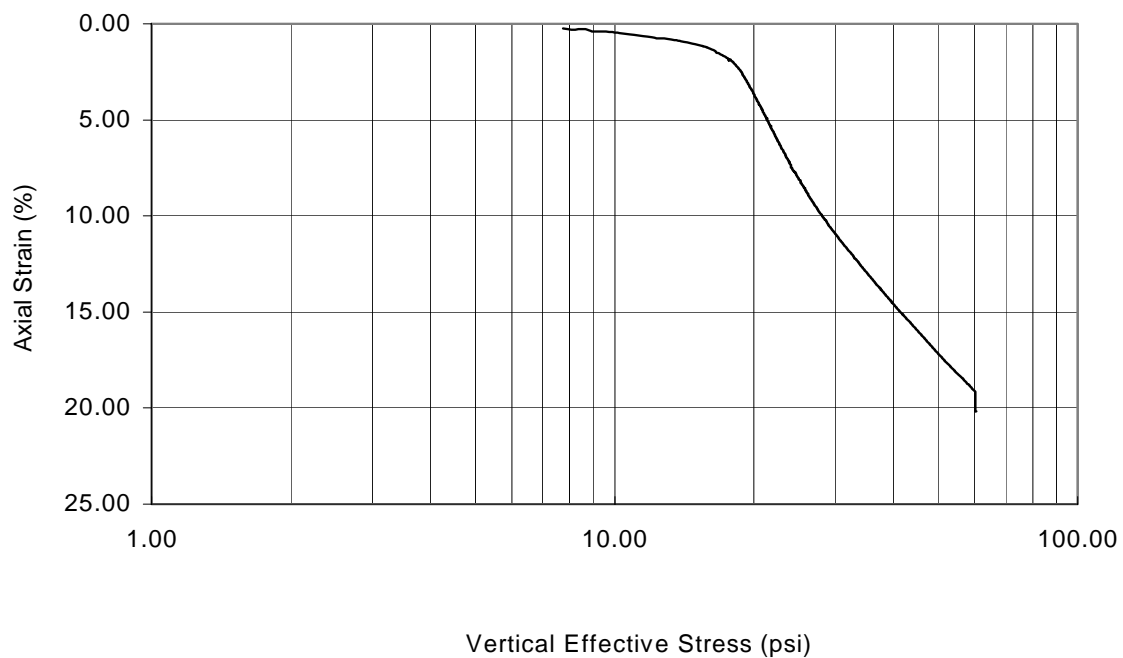


Figure 1.12. Consolidation curve for a sample from boring RS-3, depth of 18.3 ft.  $K_0$  consolidated to  $\sigma'_v$  equal to 2.5 times  $\sigma'_{v0}$ , which is 60 psi. OCR=1.

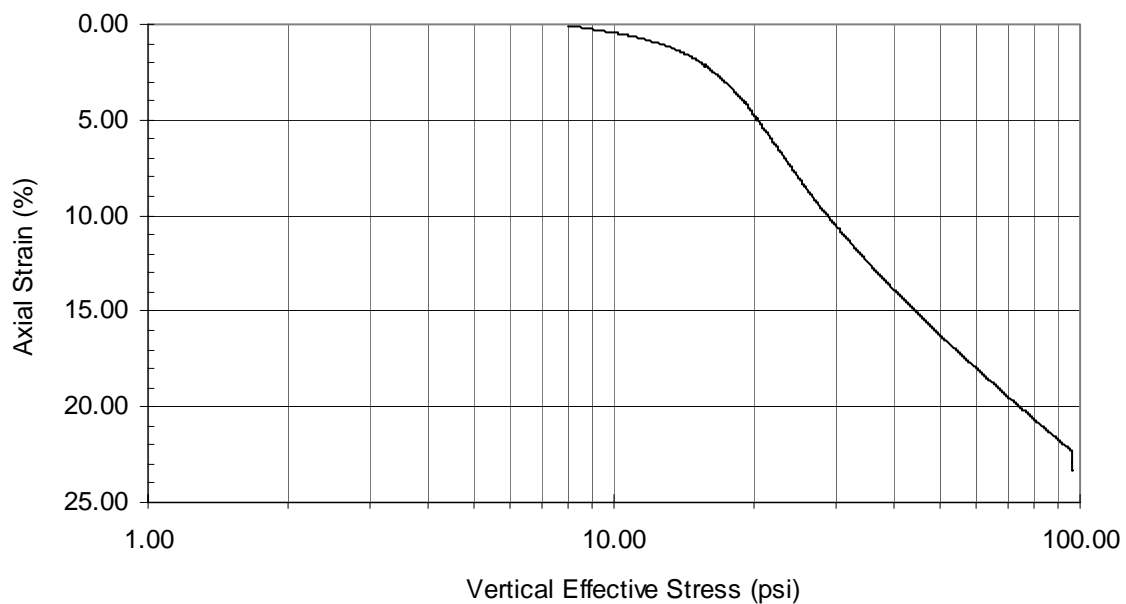


Figure 1.13. Consolidation curve for a sample from boring HF-2, depth of 18.6 ft.  $K_0$  consolidated to  $\sigma'_v$  equal to 4.0 times  $\sigma'_{v0}$ , which is 96 psi. OCR=1.

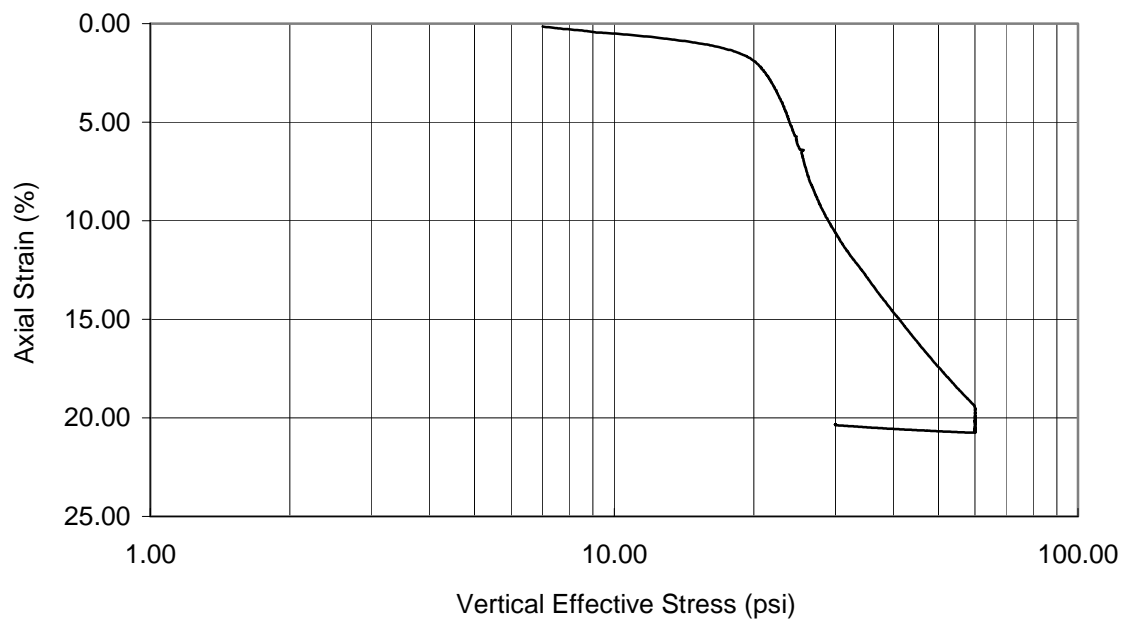


Figure 1.14. Consolidation curve for a specimen from boring RF-4 at 18.0 ft.  $K_0$  consolidated to  $\sigma'_{vm}$  of 60 psi, and then allowed to swell to 30 psi for an OCR of 2.0.

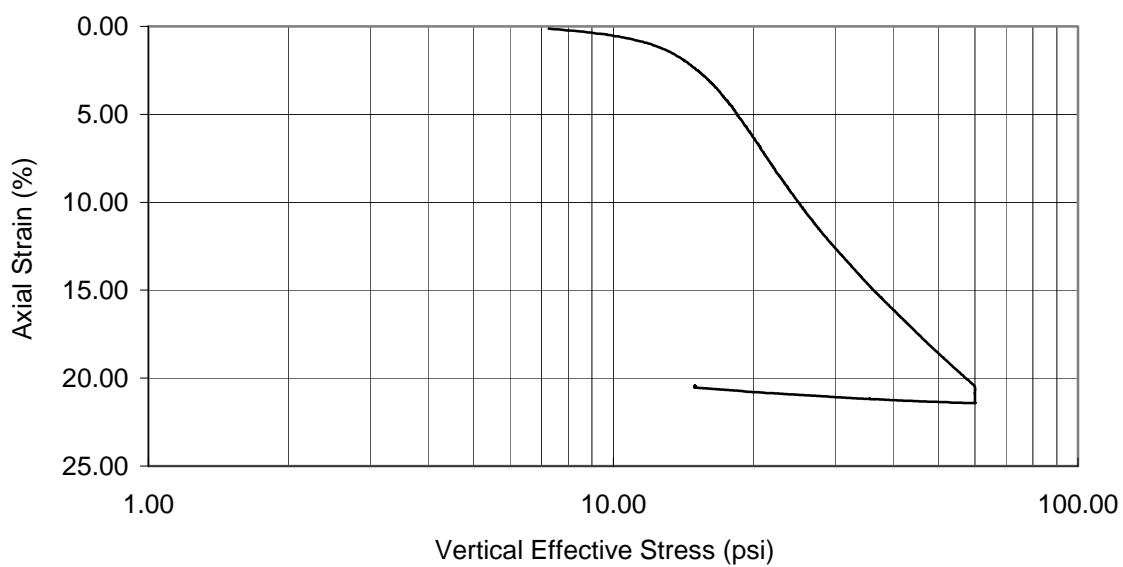


Figure 1.15. Consolidation curve for a specimen from boring RF-4 at 18.2 ft.  $K_0$  consolidated to  $\sigma'_{vm}$  of 60 psi, and then allowed to swell to 15 psi for an OCR of 4.0.

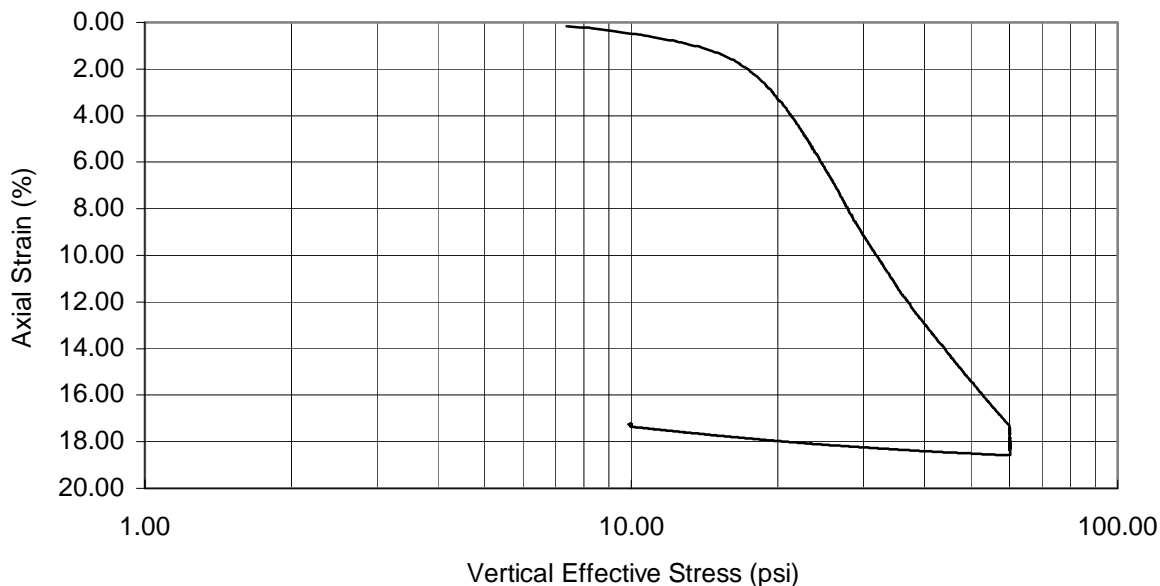


Figure 1.16 Consolidation curve for a specimen from boring RF-4 from a depth of 18.6 ft.  $K_0$  consolidated to  $\sigma'_{vm}$  of 60 psi, and then allowed to swell to 10 psi for an OCR of 6.0.

The Casagrande constructions on each of these plots indicate a preconsolidation pressure that is approximately 25% less than those seen in the CRS portion of this testing program. This rate sensitivity is not an unexpected result. In fact, the 25% reduction occurring as a result of a reduction in strain rate from 2.25% per hour to 0.3% per hour, closely resembles the shift seen in a parametric study done on the Bonneville deposit in a different area. The shift seen in that study was on the order of 30% per log cycle.

A noticeable exception to this rate sensitivity is the curve from the first overconsolidated shear test, as shown in Figure 1.14. The very distinct break around the preconsolidation pressure, and the exaggerated “S” shape in the consolidation curve suggests that this sample be of the highest quality. The combination of mud rotary drilling techniques and fixed piston sampling techniques would generally be expected to

provide the least disturbed specimens of any of the techniques used for this study. The reader is referred to the sample disturbance report (Bay et al., 2003) for a more detailed explanation of the various drilling and sampling combinations, and their effect on sample disturbance.

#### 1.4.3 Shearing Phase Results from Triaxial Tests

The consolidation phase for each specimen is followed by a period of creep. In the normally consolidated tests, this creep period was approximately 24 hours. In the overconsolidated tests it was necessary to have two periods of creep: one following the initial consolidation, and another following the swelling to a given overconsolidation ratio. These periods of creep allow dissipation of any pore pressures gradients created during either the consolidation or swelling phase for the specimen. When pore pressures are effectively eliminated shearing is initiated.

Tables 1.2 and 1.3 summarize the important results from the  $\overline{CK_0U}$  portion of the testing program. The  $\sigma'_{v0}$  column displays the final pressure the specimen was exposed to immediately prior to shearing. The  $\sigma'_{vm}$  column displays the maximum consolidation pressure the specimen was exposed to during the consolidation phase of the testing. The OCR column is the ratio of the  $\sigma'_{vm}$  column to the  $\sigma'_{v0}$  column, this ratio being the overconsolidation ratio. The  $A_f$  column lists Skempton's A value at failure for each of the specimens. The  $S_u/\sigma'_{vm}$  column lists the ratio of maximum compressive shear strengths to consolidation pressure measured just prior to shearing. All of the trends seen in Table 1.2 give plots that are very similar to those given in Koutsoftas (1986) study on

Table 1.2. Results summary for the  $\overline{CK_0U}$  testing, for samples from the 17-19 ft depth.

Boring	$\sigma'_{v0}$ (psi)	$\sigma'_{vm}$ (psi)	OCR	$A_f$	$S_u/\sigma'_{vm}$	$\phi'$ (deg)*	$E_{50}$ (psi)
HS-1	36	36	1.0	1.55	.317	26.99	3388
RS-3	60	60	1.0	1.58	.326	27.08	6125
HF-2	96	96	1.0	1.45	.315	26.36	6328
RF-4	30	60	2.0	.363	.595	32.29	4310
RF-4	15	60	4.0	.134	.989	34.27	2202
RF-4	10	60	6.0	0.04	1.398	36.18	1350

\* assuming  $c' = 0$ .

marine clays. Many of the plots used to represent this data are presented in the style of Koutsoftas, to facilitate comparison.

Figures 1.17 through 1.22 show the shear stress vs. strain data for each of the tests performed. This data is collected in real time during the shear phase of the test.

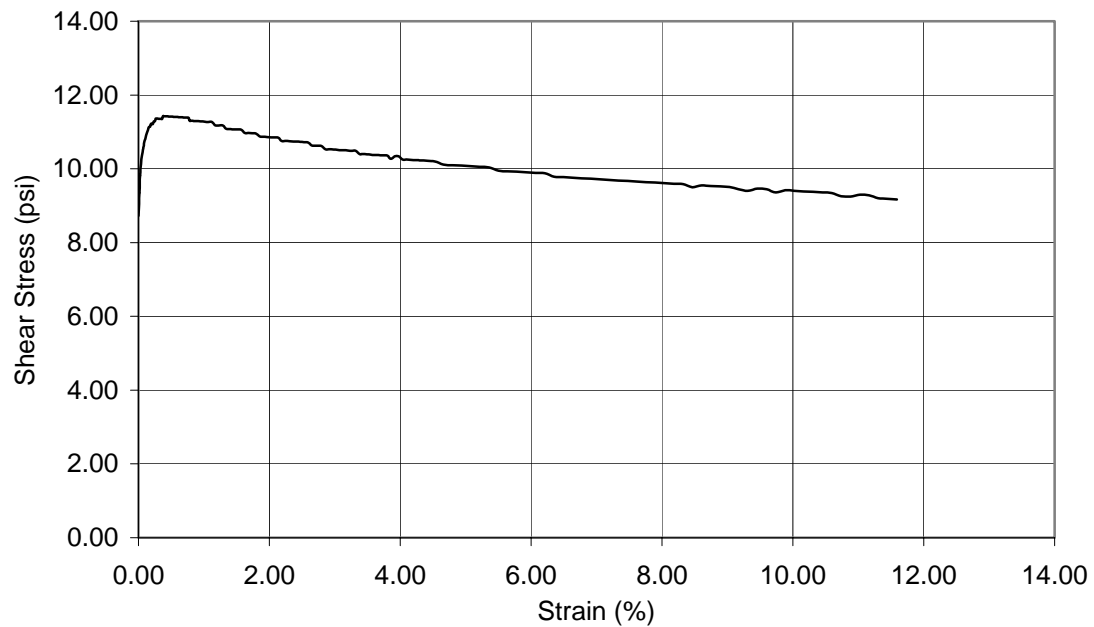


Figure 1.17. Shear Stress vs. Strain for the  $1.5 \times \sigma'_{v0}$ , Normally Consolidated Specimen.

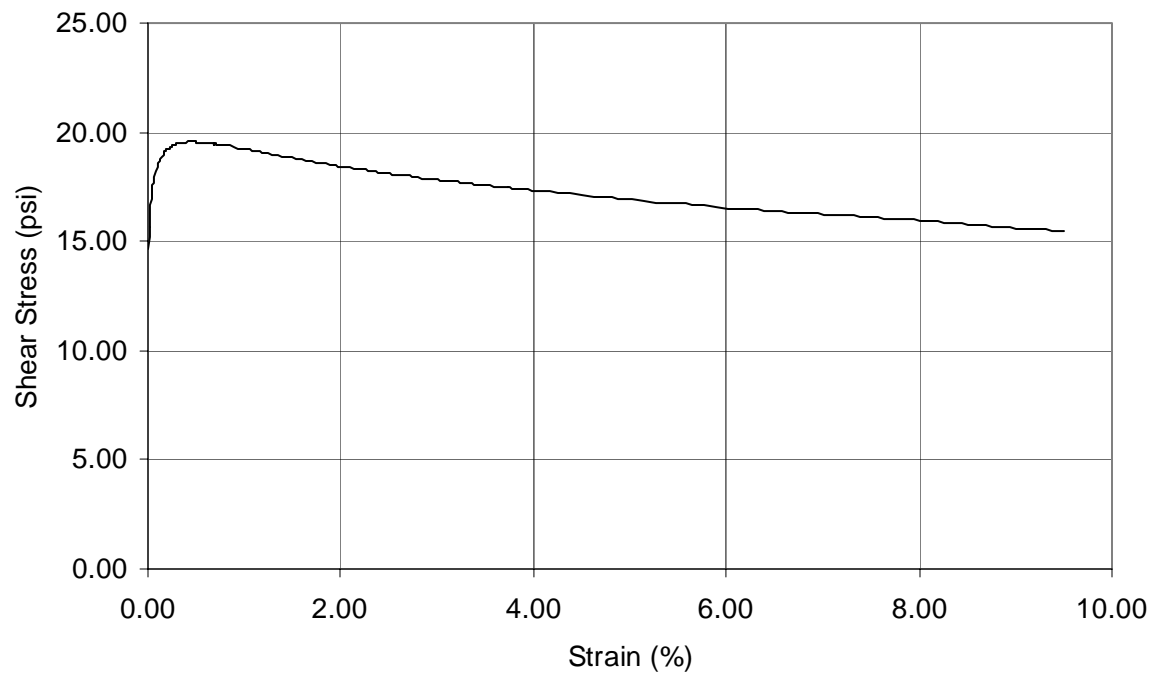


Figure 1.18. Shear Stress vs. Strain for the  $2.5 \times \sigma'_{v0}$  Normally Consolidated Specimen.

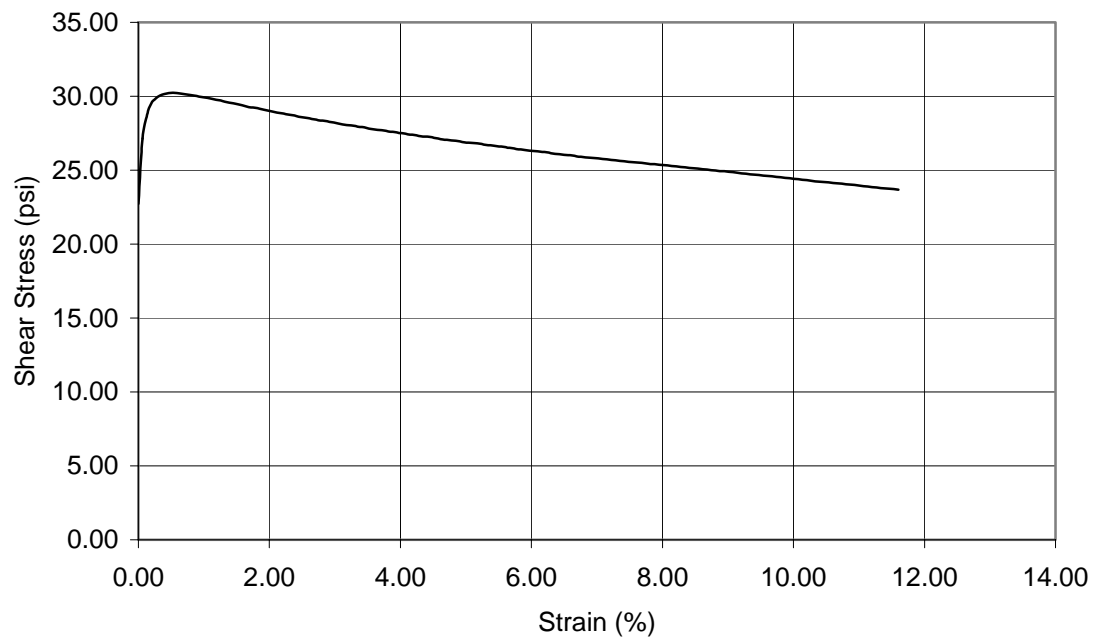


Figure 1.19. Shear Stress vs. Strain for the  $4.0 \times \sigma'_{v0}$  Normally Consolidated Specimen.

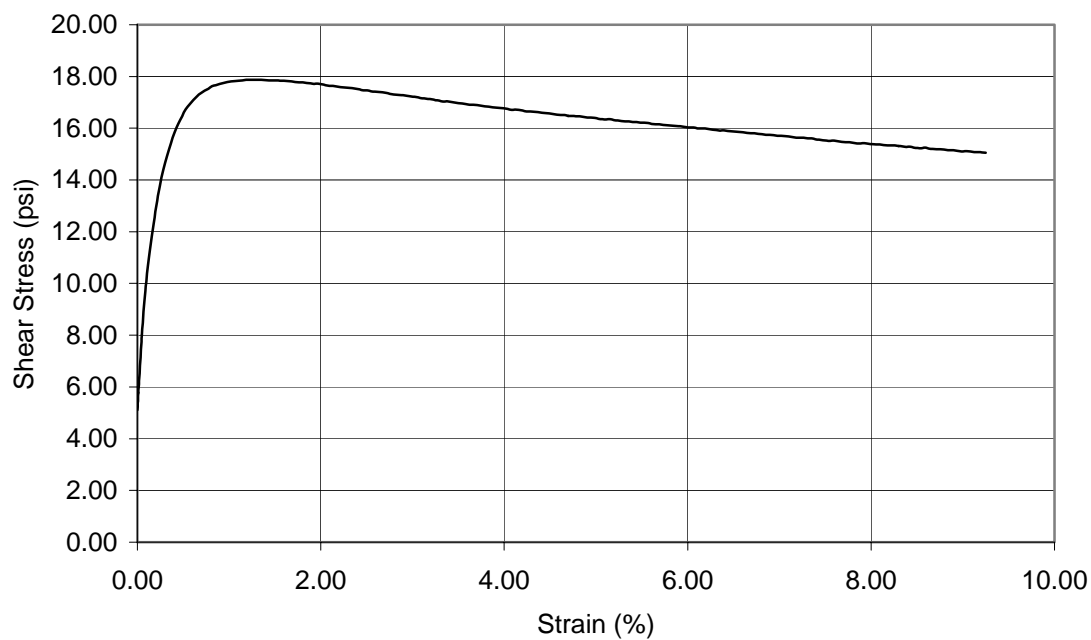


Figure 1.20. Shear Stress vs. Strain for the OCR = 2.0 Overconsolidated Specimen.

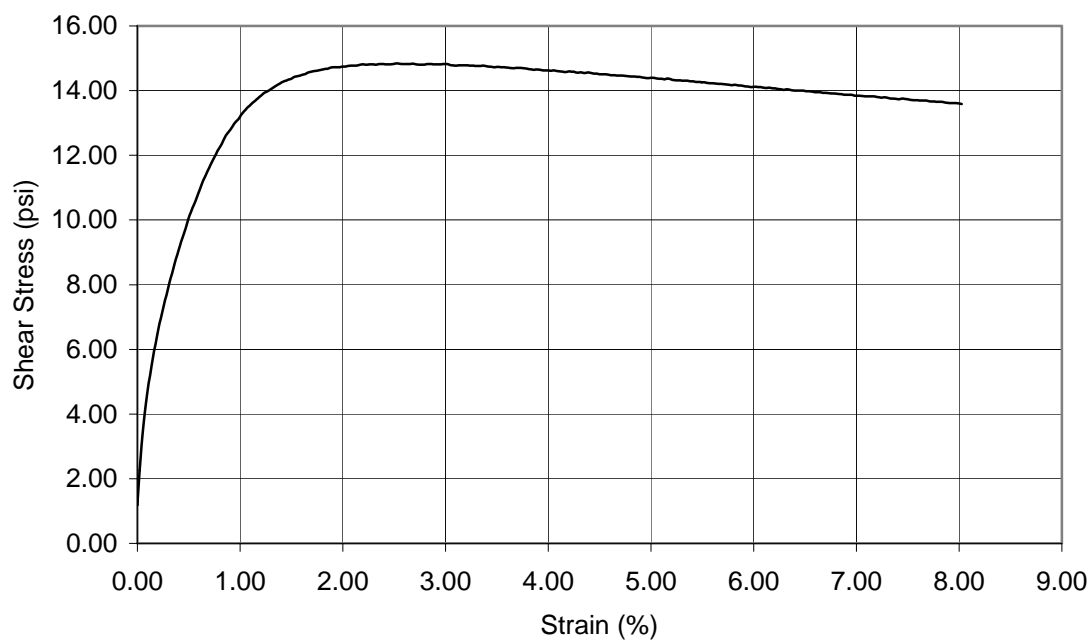


Figure 1.21. Shear Stress vs. Strain for the OCR = 4.0 Overconsolidated Specimen.

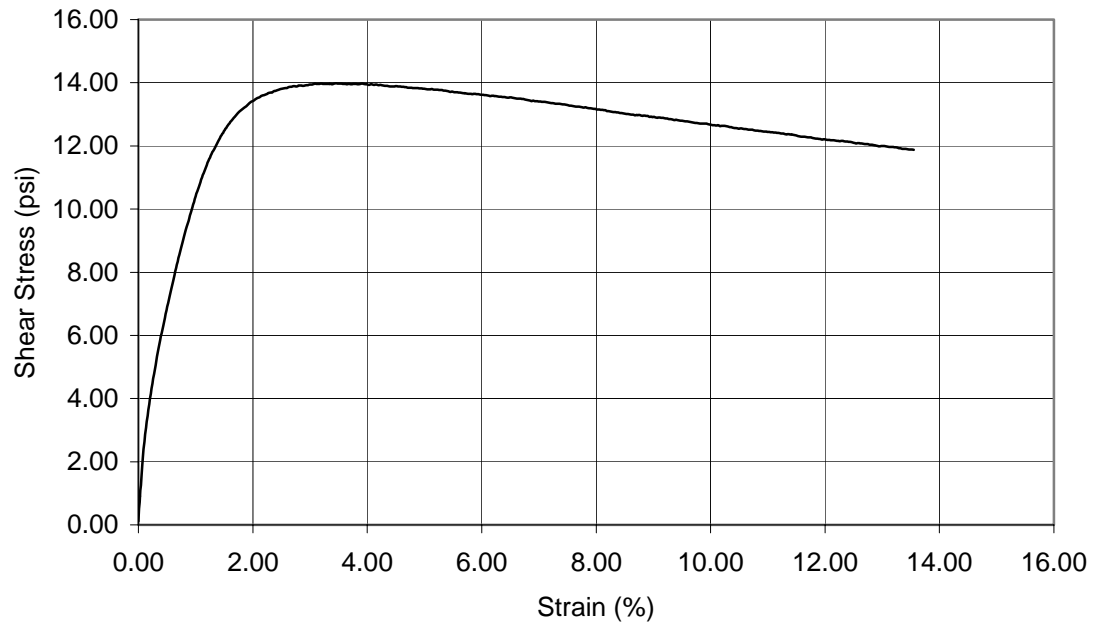


Figure 1.22. Shear Stress vs. Strain for the OCR = 6.0 Overconsolidated Specimen.

Another observation to be made from this portion of the triaxial testing is the increase in failure strain with OCR. The reduction of strain softening with increasing OCR mirrors that seen in Koutsoftas, (1986). This data is plotted in Figure 1.23. The reason for this apparent increase in the failure strain appears to be directly related to the amount of swelling allowed to create samples at discrete overconsolidation ratios. Overconsolidated specimens will compress elastically without failing during the shear phase. For any specimen, greater swelling would allow greater elastic compression prior to failure. A specimen with an OCR of six would naturally have undergone greater swelling than a specimen with an OCR of two. It follows that this same specimen with an OCR of six would experience greater elastic compression prior to failure in triaxial compression shear strength testing.

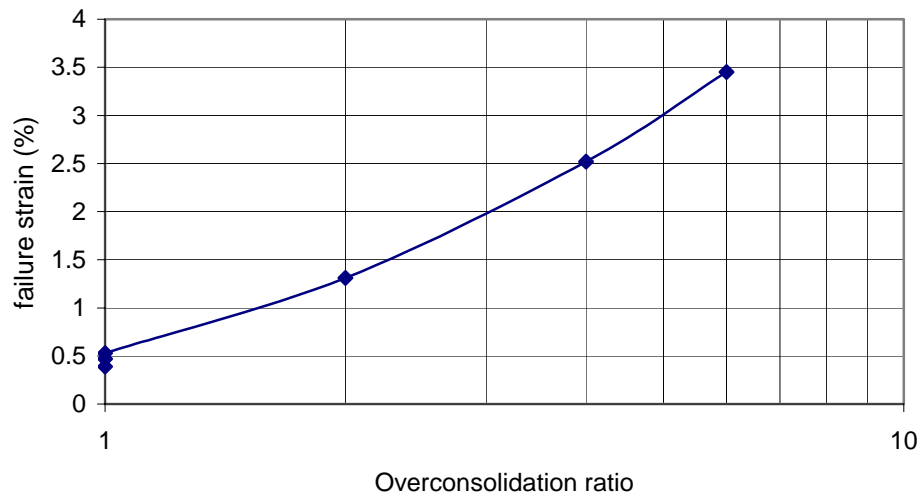


Figure 1.23. Increasing Failure Strain vs. OCR, for the  $\overline{CK}_0U$  Specimens from 17-19 ft.

#### 1.4.4 SHANSEP Undrained Shear Strength Ratios

The stated objective of this study was to develop Stress History and Normalized Soil Engineering Properties, (SHANSEP), or Normalized Soil Parameters (NSP). More specifically the stated goal was to calculate the exponent  $m$  as seen in equation 1.1 (Ladd and Foote, 1974).

$$s_u/\sigma'_{v0} = 0.32(OCR)^{0.82} \quad (1.2)$$

where

$S$  is the normally consolidated ratio of  $s_u/\sigma'_{vc}$  ( $S=0.32$ ),

$s_u/\sigma'_{v0}$  is the shear strength “normalized” with respect to the in situ vertical stress,

OCR is the overconsolidation ratio, and

$m$  is an exponent established by curve fitting ( $m=0.82$ ).

Figure 1.24 shows the normalized plot of this equation, and the results of a logarithmic curve fitting function to produce a value of  $m$  of 0.82 for the data collected in this study. Koutsoftas reports  $m$  values from 0.80 to 0.85, for his work. The plot in Figure 1.25 is a plot of normalized shear strength vs. overconsolidation ratio from  $\overline{CK_0U}$  triaxial compression for the I-15 clays and for the marine clay tested in the Koutsoftas study.

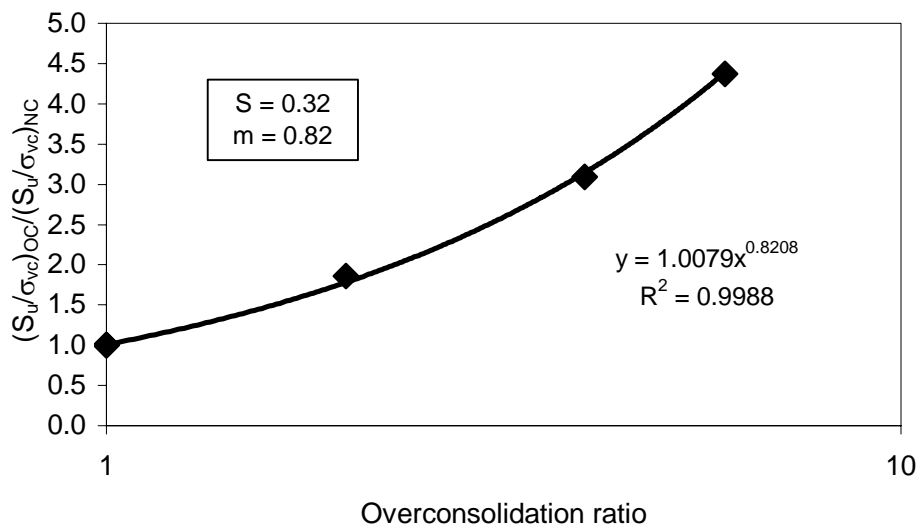


Figure 1.24. Relative increase in undrained strength ratio with increasing OCR.

Plotted in Figure 1.25 are results from Koutsoftas (1986), from work done on silty clays collected off the coast of New Jersey. Koutsoftas describes the specimens for his test as, “an inorganic marine clay with liquid limits between 25 and 45 percent and a plasticity index of  $18 \pm 5$  percent. The Atterberg limits plot slightly above the A-line of the plasticity chart.” Comparison of the results of the indices for this study and those given by Koutsoftas (1986) demonstrates a close similarity between this marine clay and

the Bonneville deposit. That the normalized shear strength values plot almost directly on top of each other demonstrate further similarity.

Despite the slight variations, the trend of increasing normalized shear strength with increasing OCR is surprisingly consistent. This consistency has positive implications for the use of the NSP approach to design. The consistency of the normalized plots for similar soils suggests that actual intrinsic soil properties are being measured, rather than method dependant properties.

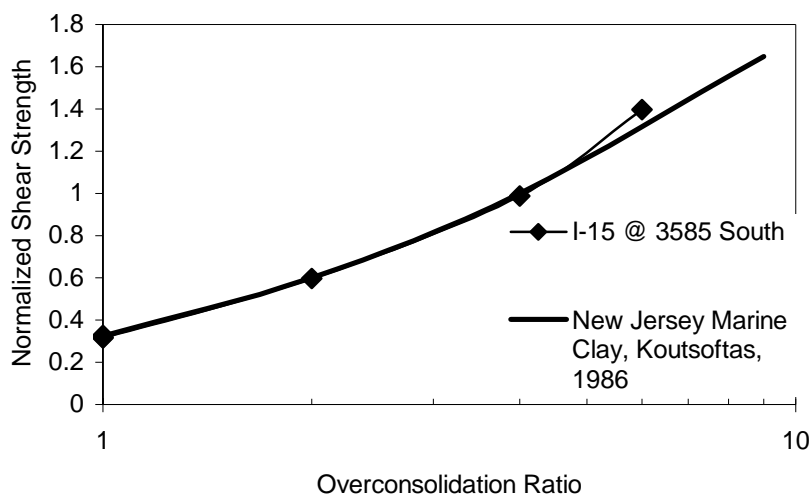


Figure 1.25. Normalized Shear Strength vs. Overconsolidation Ratio.

Table 1.3. Summary of Results for  $\overline{CK}_0U$  Triaxial Compression Tests.

Boring	Depth (ft)	$\sigma_{v \max}$ (psi)	$\sigma_{v0}$ (psi)	OCR	Strain $\epsilon_f$	$E_{u50}$ (psi)	$S_u$ (psi)	$S_{u \text{ normal}}$	$E_{u50}/S_u$	$E_{u50}/\sigma_{v0}$	$A_f$	$\frac{S_{u \text{ normal OC}}}{S_{u \text{ normal NC}}}$	$\phi$
HS-1	17-19	36	36	1	0.39	3388	11.43	0.318	296.4	94.11	1.55	1	26.99
RS-3	17-19	60	60	1	0.47	6125	19.57	0.326	313	102.1	1.58	1	27.08
HF-2	17-19	96	96	1	0.53	6328	30.24	0.315	209.3	65.92	1.45	1	26.36
RF-4	17-19	60	30	2	1.31	4310	17.87	0.596	241.2	143.7	0.36	1.863	32.29
RF-4	17-19	60	15	4	2.52	2202	14.84	0.989	148.4	146.8	0.13	3.092	34.27
RF-4	17-19	60	10	6	3.45	1350	13.98	1.398	96.57	135	0.04	4.369	36.18

## 1.5 Conclusions

The purpose of this project was to determine the Stress History and Normalized Engineering Properties (SHANSEP) parameters to characterize the undrained shear strength of soft Bonneville clay. Soil samples for this work were obtained near the MSE retaining wall near 3600 South on I-15 in Salt Lake City. The soil samples were obtained from a very soft clay layer between 18 and 20 ft deep. A series of constant rate of strain (CRS) consolidation tests and  $K_0$  consolidated undrained triaxial shear tests ( $\overline{CK_0U}$ ) were performed to determine these SHANSEP parameters.

Undrained shear strength in clays is a function of the soil type and structure, water content, stress history (over-consolidation ratio (OCR) and consolidation condition), and stress path during undrained loading. Classical analyses do not account for the effects of stress history and stress path in characterizing soil strength and in predicting field behavior. Stress history and stress path have very large effects on undrained strength of clays, leading to large errors in classical undrained analyses.

One approach, which accounts for the effects of stress history and stress path is the SHANSEP approach. The general idea behind the SHANSEP method is to perform a series of laboratory tests, which carefully control the stress conditions during consolidation, and control the stress path during undrained shear. These tests are performed over a range of stress histories and stress paths. The in situ stress history of the soil is then evaluated, and the stress path to which the soil will be imposed is determined. Then, strengths from the laboratory tests, which most closely replicate the field conditions, are used to predict the field behavior.

In the SHANSEP approach the following equation is used to describe the undrained shear strength of a soil subjected to a particular stress path:

$$\frac{S_u}{\sigma'_{vo}} = S \times (\text{OCR})^m,$$

where:  $S_u$  is the undrained shear strength,

$\sigma'_{vo}$  is the in situ effective vertical stress,

$S$  is the normally consolidated ratio of  $\left(\frac{S_u}{\sigma'_{vo}}\right)_{nc}$ ,

OCR is over consolidation ratio, and

$m$  is an exponent that usually falls between 0.75 and 1.0.

From this work, the following equation was found to predict the undrained shear strength of Bonneville clay in triaxial compression:

$$\frac{S_u}{\sigma'_{vo}} = 0.32 \times (\text{OCR})^{0.82}.$$

These results are based upon CK<sub>0</sub>U triaxial compression tests performed at OCR's from 1 to 6. These values of SHANSEP parameters are consistent, and in the range of values reported by other investigators for similar soils. The undrained shear strength for triaxial compression provides a close, but slightly conservative, estimation of the undrained shear for soils in a plane-strain, active condition.

The results of these tests were very consistent, and it was observed that normalized parameters very accurately describe the undrained shear strength and deformation behavior of these Bonneville clays. This indicates that SHANSEP analyses

will provide good predictions of undrained field behavior, and will provide improved predictions of undrained soil behavior over classical approaches.

This work shows that the SHANSEP approach works well for Bonneville clays. Bonneville clays appear to have good normalized behavior (the undrained strength is proportional to confining pressure) and show consistent effects of overconsolidation ratio.

The SHANSEP undrained strength parameters determined in this work can be used by UDOT and other designers for preliminary analyses. For instance, these strength parameters could be used in a preliminary analysis of an embankment on a Bonneville clay foundation to determine if the embankment would be stable if constructed to full height, or if staged construction is required. Final design should be based upon additional laboratory testing of soils from the construction site.

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## APPENDICES

Appendix A  
Laboratory Results

Table A.1 Laboratory results from boring HS-1

Depth (ft)	$\sigma'_v$ (psi)	$\sigma'_p$ (Casagrande) (psi)	$\sigma'_p$ (Modulus) (psi)	$\sigma'_p$ error band (psi)	m	$w_n$ (%)	PL (%)	LL (%)	$C_{CE}$	$\gamma_t$ (pcf)	$\gamma_a$ (pcf)	Grain Size %>75mm	Grain Size %2-75mm	Grain Size %<2mm
9.5-11.5	7.5	64.0		19-107			19	32	.135	120.9				31.0
12-14	10.0	33.0	23.0	15-53	12.2		18	26.5	.134					38
14.5-16.5	10.8	49.0	39.0	28-70	13.7	31	22.7	34	.162	119	90.5	.4	64.6	35.0
17-19	11.8	23.0	24.0	22-24	9.10	58	36	26	0.50	101.6	62.7	3.3	61.7	35.0
19.5-21.5	12.4													
22-24	13.6	25.0	30.0	12-38	14.6	36.4	23.4	31.7	.139	115.6	84.1			33.0
24.5-26.5	14.4	22.0	31.0	16-46	17.3	26.4	18.2	22.3	.129	124.4	97	17.9	62.1	20.0
27-29	15.7	36.0	33.0	18-46	19.6				.112	124.8	98.6	13.2	67.8	19.0
29.5-31.5														
32-34						17.7								
34.5-36.5	19.1						20.0	26.4				26.0	51.0	23.0

Table A.2 Laboratory results from boring HF-2

Depth (ft)	$\sigma'_v$ (psi)	$\sigma'_p$ (Casagrande) (psi)	$\sigma'_p$ (Modulus) (psi)	$\sigma'_p$ error band (psi)	m	$w_n$ (%)	PL (%)	LL (%)	$C_{CE}$	$\gamma_t$ (pcf)	$\gamma_d$ (pcf)	Grain Size %>75m m	Grain Size %2- 75mm	Grain Size %<2mm
9.5-11.5		53.0	45.0	30-98	11.5	30.3	21.9	31.6	0.156	118.8	90.4	0.1	59.9	40.0
12-14		30.0	32.0	20-41	13.4	33.2	21.8	30.0	0.159	117.8	88.2	1.1	78.9	20.0
14.5-16.5		35.0	34.0	22-42	14.4	27.8	19.0	30.5	0.132	119.1	90.9	0.6	69.4	30.0
17-19		20.0	34.0	12-26	12.0	48.7	23.9	37.0	0.269	101.2	62.5	1.1	60.9	38.0
19.5-21.5														
22-24														
24.5-26.5														
27-29						27.3	18.3	23.7						
29.5-31.5						27.1	16.0	21.0				25.0	48.0	27.0
32-34														
34.5-36.5						23.2						8.8	56.2	35.0

Table A.3 Laboratory results from boring RS-3

Depth (ft)	$\sigma'_v$ (psi)	$\sigma'_p$ (Casagrande) (psi)	$\sigma'_p$ (Modulus) (psi)	$\sigma'_p$ error band (psi)	m	w <sub>n</sub> (%)	PL (%)	LL (%)	C <sub>CE</sub>	$\gamma_t$ (pcf)	$\gamma_d$ (pcf)	Grain Size %>75m	Grain Size %2-75mm	Grain Size %<2mm
9.5-11.5	7.5													
12-14	10.0	37.0	35.0	22-55	13.7	33.4	22.9	31.6	0.138	118.5	89.8	0.7	61.7	38.0
14.5-16.5	10.8	43.0	44.0	30-55	13.7	32.6	22.6	32.5	0.170	119.5	90.7	0.6	69.4	30.0
17-19	11.8	22.0	21.0	17-24	12.0	67.0			0.419	101.1	81.0	5.2	60.8	34.0
19.5-21.5	12.4											51.0	28.0	21.0
22-24	13.6													
24.5-26.5	14.4													
27-29	15.7													
29.5-31.5														
32-34														
34.5-36.5	19.1					21.3	18.5	23.0				10.2	57.2	32.0

Table A.4 Laboratory results from boring RF-4

Depth (ft)	$\sigma'_v$ (psi)	$\sigma'_p$ (Casagrande) (psi)	$\sigma'_p$ (Modulus) (psi)	$\sigma'_p$ error band (psi)	m	$w_n$ (%)	PL (%)	LL (%)	$C_{CE}$	$\gamma_t$ (pcf)	$\gamma_d$ (pcf)	Grain Size %>75m	Grain Size %2-75mm	Grain Size %<2mm
9.5-11.5	7.5													29.0
12-14	10.0	45.0	47.0	27-70	12.6				0.136					
14.5-16.5	10.8	55.0	57.0	31-70	10.6	28.3	23.8	31.5	0.129	119.7	94.3	0.9	59.6	39.5
17-19	11.8	28.0	31.0	22-31	13.0	58.4	23.5	47.0	0.490	102.4	63.2	0.9	62.1	37.0
19.5-21.5	12.4													
22-24	13.6	21.0	25.0	11.3-37	19.4	32.3	18.3	24.7	0.093	123.5	105.4	77.2	47.2	30.0
24.5-26.5	14.4	35.0	43.0	22-46	15.6	27.1	16.7	26.5	0.126	122.5	95.4	5.3	55.7	39.0
27-29	15.7													
29.5-31.5														
32-34														
34.5-36.5	19.1						18.5	23.0						